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Estimating the Deformability and Strength of Rock Masses-In-Situ Tests and Other Procedures

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ABSTRACT

This report was prepared for presentation at the STRATCOM Advanced Concept Technology Demonstration (ACTD) meeting held in Albuquerque, New Mexico, May 21, 2003.

It discusses the methods that can be used to estimate the mechanical properties of rock masses, such as deformability and strength. Special emphasis is put on the fact that rock mass properties are subject to an effect of scale, i.e. the properties measured on laboratory-scale samples are not representative of in-situ properties because of the presence of geologic discontinuities.

This information is relevant to the planning of new field tests to asses the effects of explosions in the ground that are part of the on-going ACTD.

ESTIMATING THE DEFORMABILITY AND STRENGTH OF ROCK MASSES

- IN-SITU TESTS AND OTHER PROCEDURES -

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ACTD MEETING, ALBUQUERQUE, NM

MAY 21, 2003

Contents

Part 1

Deformability of Rock Masses

Plate bearing tests on isotropic rocks

Boussinesq (1885) solution for stress distribution under a square area in an elastic isotropic medium. The stress contours give an estimation of the volume of rock exercised by the test (after Seed, 1965).

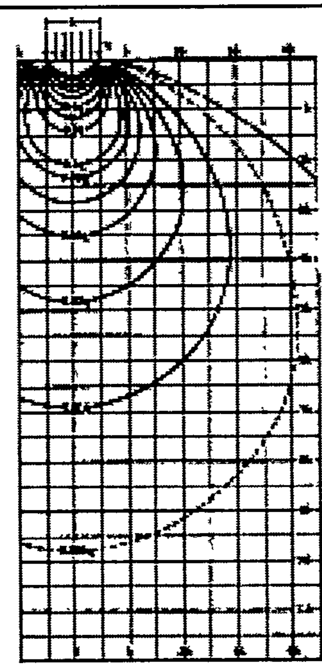
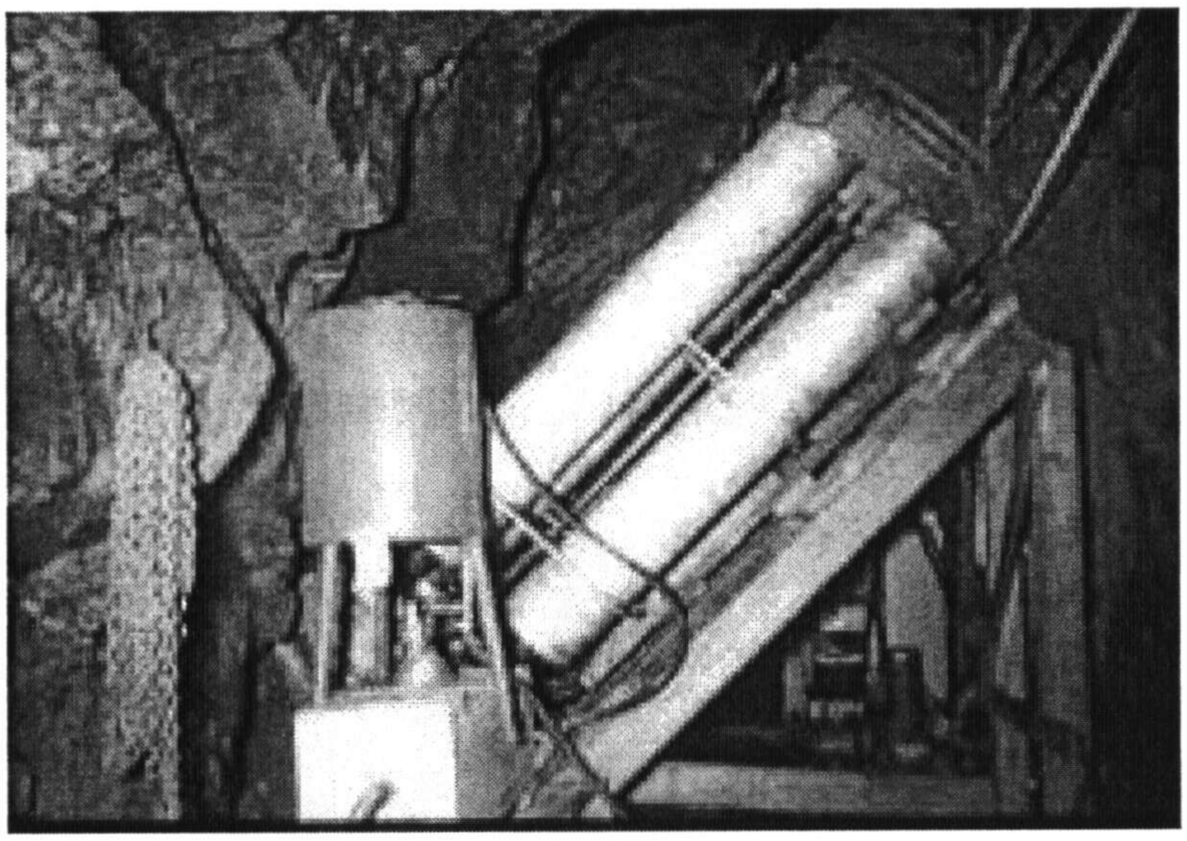


Plate tests - Example (Wallace et al, 1970)



Note: USBR cost, 10 years ago, at Monk Hollow dam site, Utah, was 300K for 6 tests, not including rock surface preparation (G. Scott, pers. communic., 05/08/03)

Plate tests - Equipment calibration (Wallace et al, 1970)

Using the U. S.Bureau of Reclamation's

4-million pound testing frame

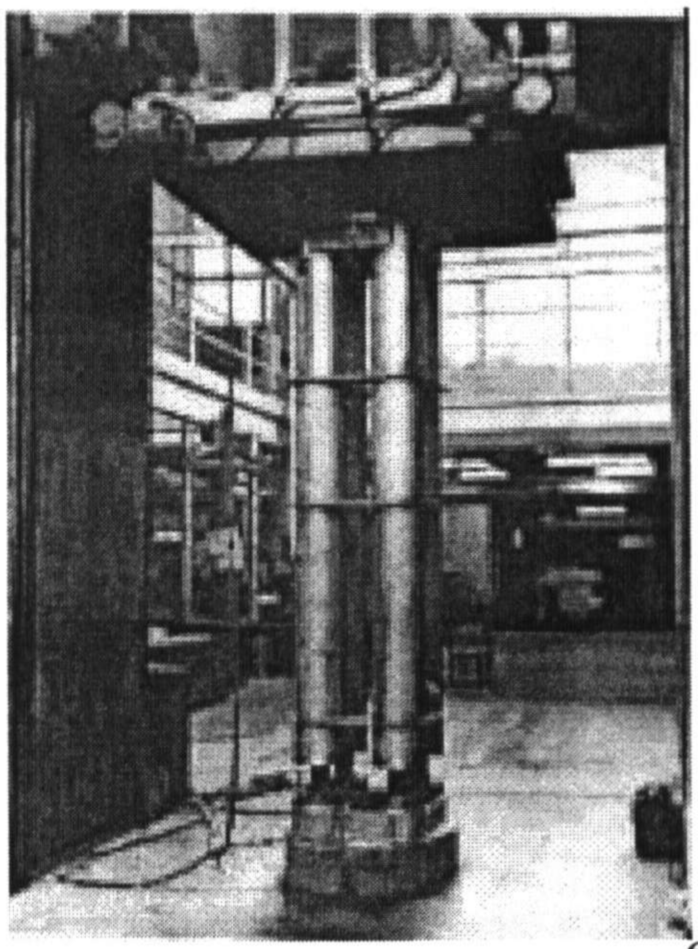


Plate test analysis (Belin, 1959)

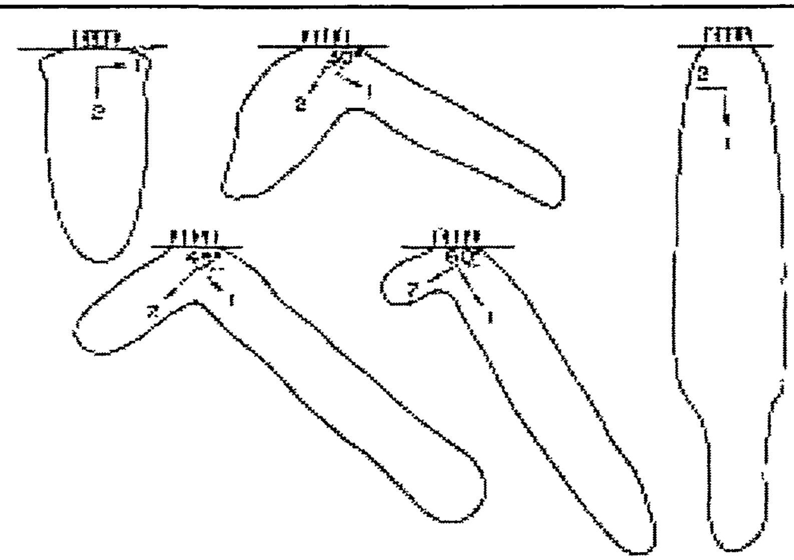
In isotropic media, the modulus of the rock mass is calculated as:

$$E = K . P . \pi . a . (1-\nu^2) / U$$

where

- K : coefficient = 0.50 for a perfectly rigid plate
  = 0.54 for a perfectly flexible plate
- P : applied pressure on the plate
- a : radius of the plate (assumed circular)
- ν : Poisson's ratio of the rock mass (assume it to be 0.25)
- U : average displacement of the plate

Plate tests on anisotropic rocks



The pressure bulb shape under a plate is influenced by rock mass anisotropy (Singh, 1973a)

Plate tests on anisotropic rocks (cont.)

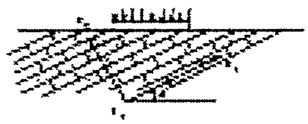
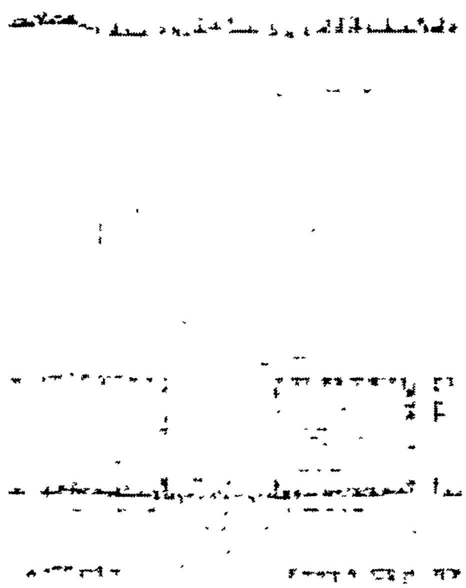


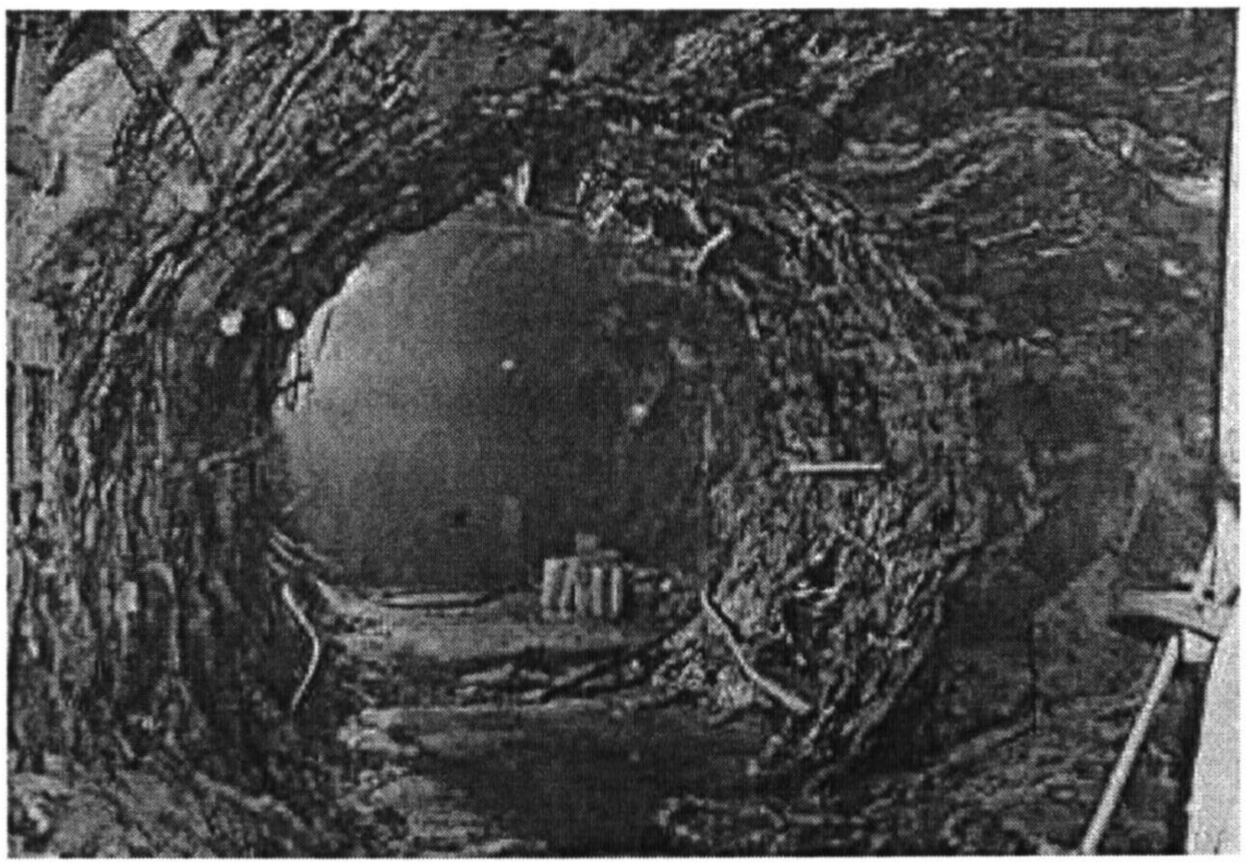
Plate shape and sensing points	E_n/E_t	Angle of anisotropy (θ)	Flexible at center	Flexible at edge	Rigid	Effect of
1	1	[illegible]	1.11	[illegible]	0.91	Plate geometry
2	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
1	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	
1	[illegible]	[illegible]	[illegible]	[illegible]	1.41	Angle of anisotropy
[illegible]	[illegible]	30	[illegible]	1.15	[illegible]	
[illegible]	[illegible]	45	[illegible]	1.47	1.47	
[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	[illegible]	

When conducting plate bearing tests on anisotropic rocks, the modulus calculated from an isotropic solution can be in error due to the rock mass anisotropy, and possibly due to the plate geometry. Results based on 2-D finite element simulations (Heuze and Salem, 1977).

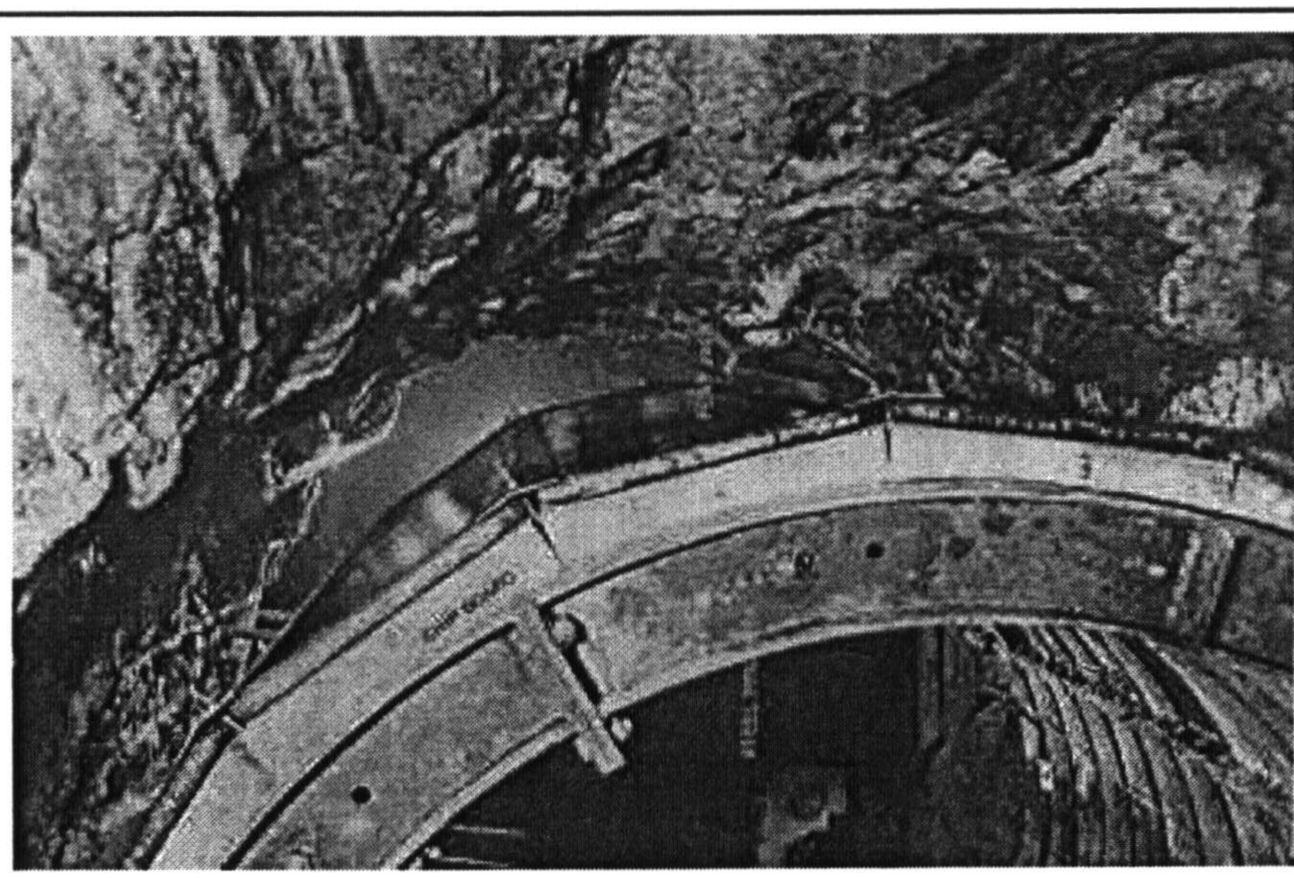
Other plate tests (Wallace et al, 1969)



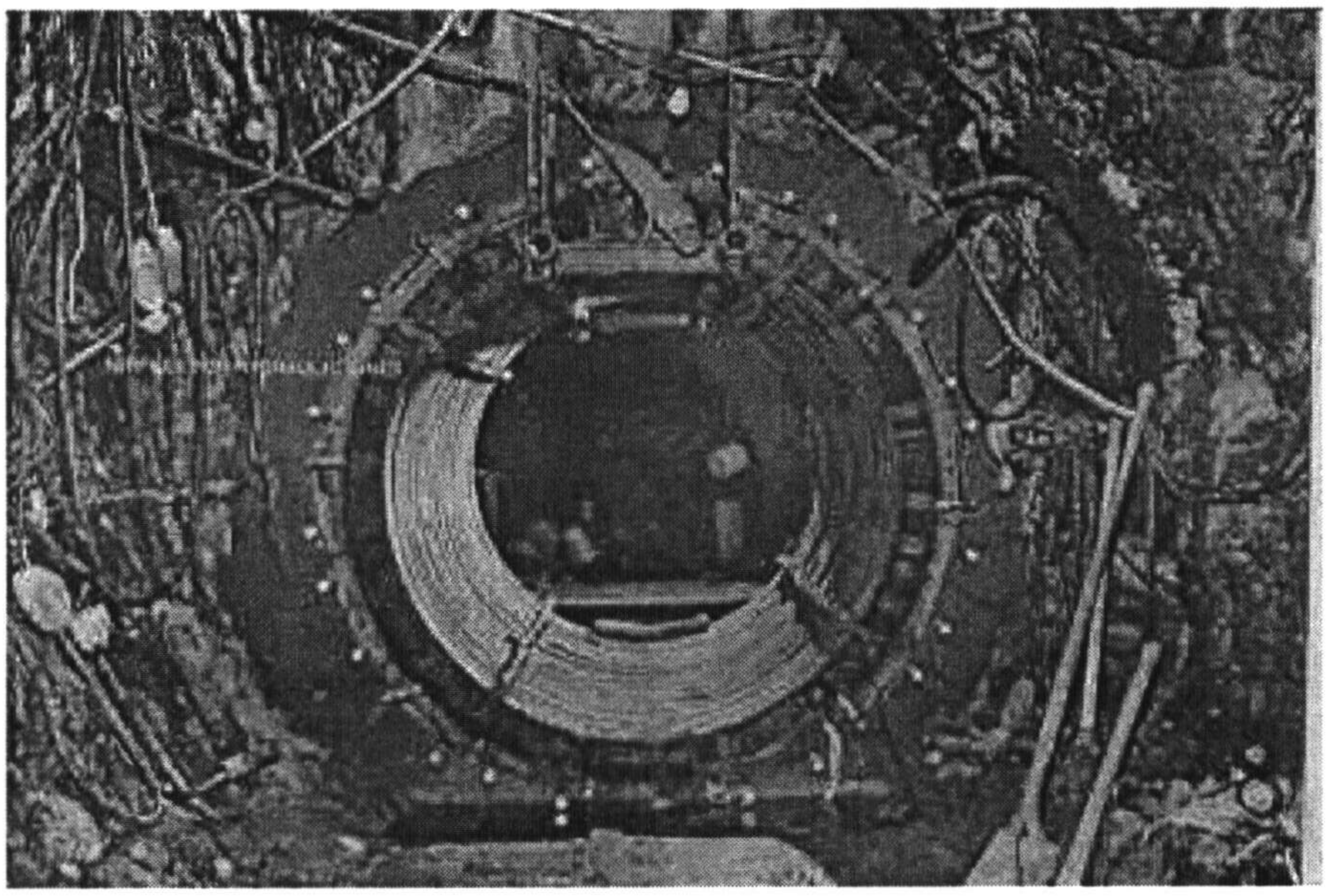
Pressure chamber tests (Wallace et al, 1970)



Pressure chamber tests (Wallace et al, 1970)



Pressure chamber tests (Wallace et al, 1970)



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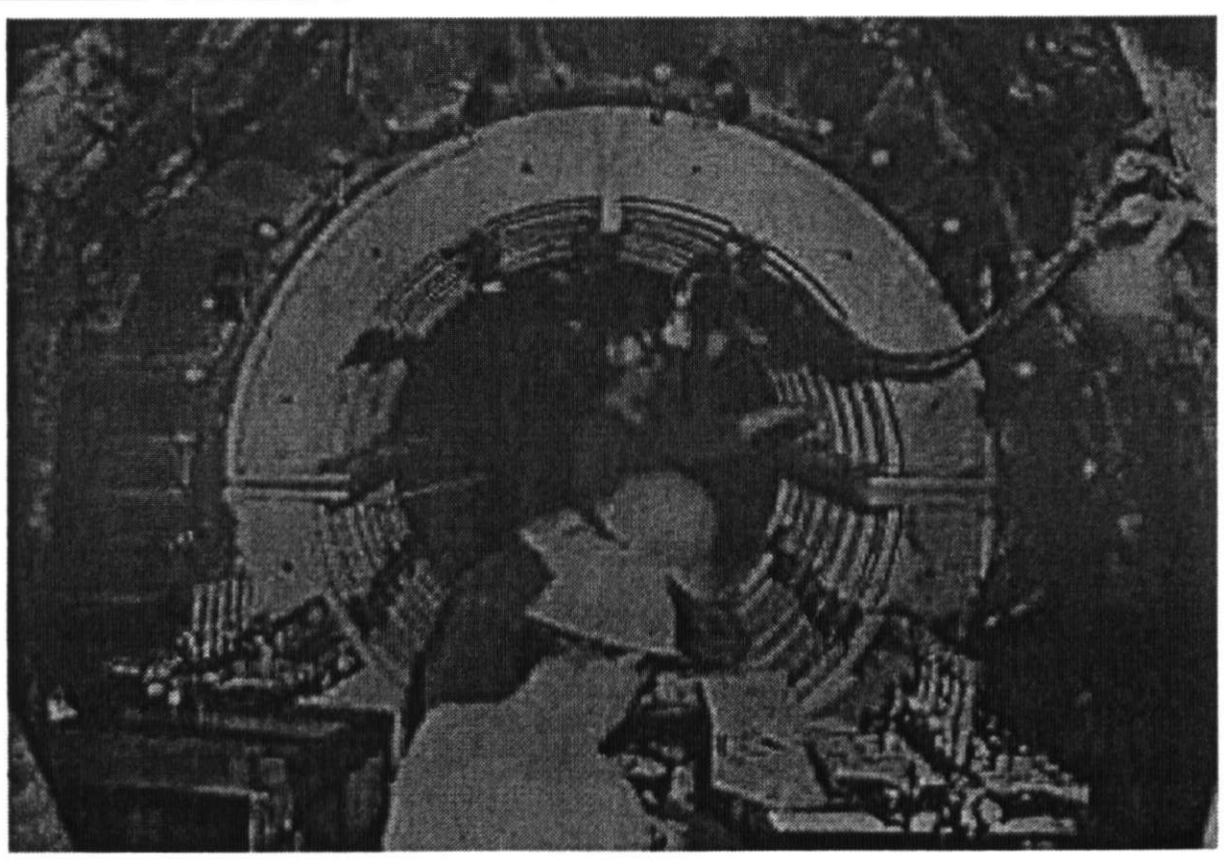
Pressure chamber tests (Wallace et al, 1970)



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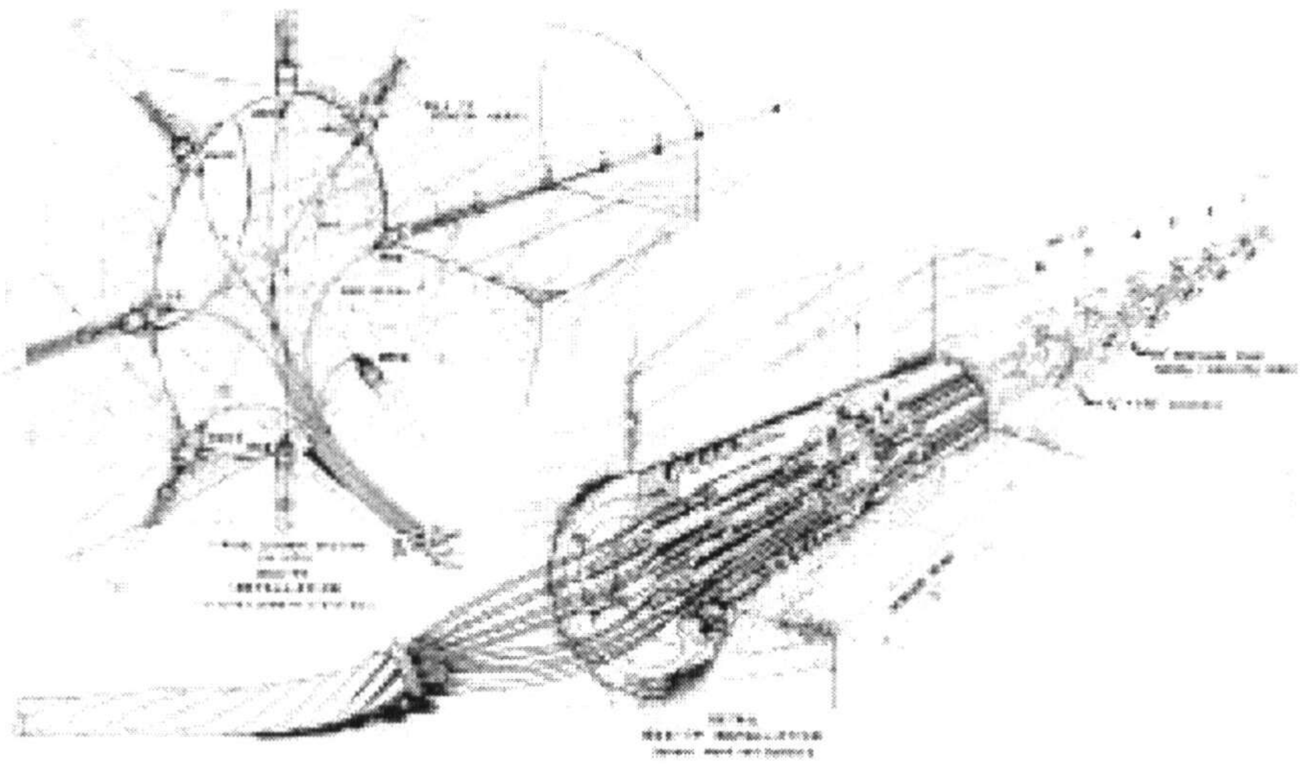
Pressure chamber tests (Wallace et al, 1970)



Multi-position extensometers are used to measure displacements inside the rock mass.

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Analysis of pressure tests in circular openings

This applies to tunnel tests such as above, or to dilatometer tests in boreholes. The rock mass modulus is obtained from:

- Measuring the change in diameter, isotropic case:

$$E = [\Delta P.\ D.(1+\nu)] / \Delta D$$

where:

ΔP : increase in applied pressure

D : diameter

ν : Poisson's ratio of the rock mass (assume 0.25)

ΔD : change in diameter

or

- Measuring the displacement U(r) at depth "r" into the rock mass:

$$E = [\Delta P\ .\ D^2.\ (1+\nu)] / [4.\ r.\ U(r)]$$

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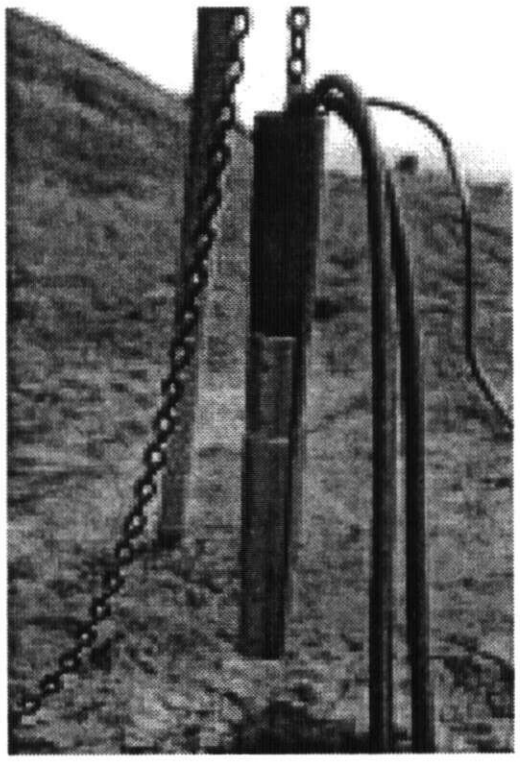
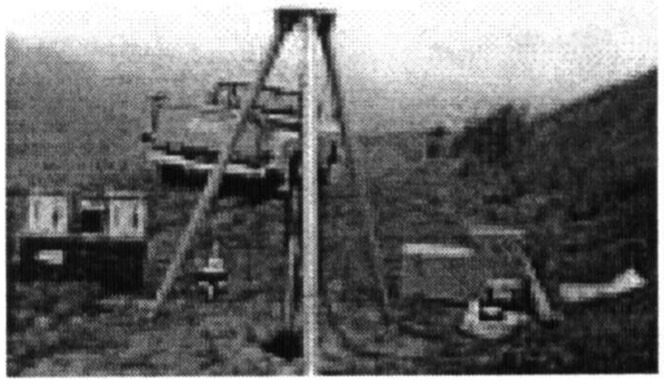
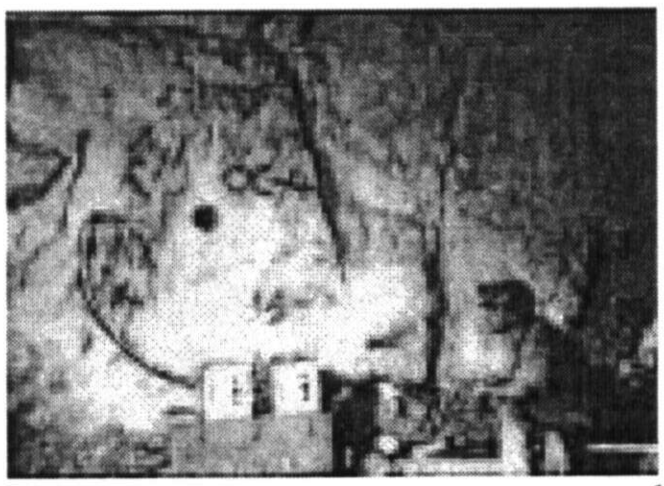
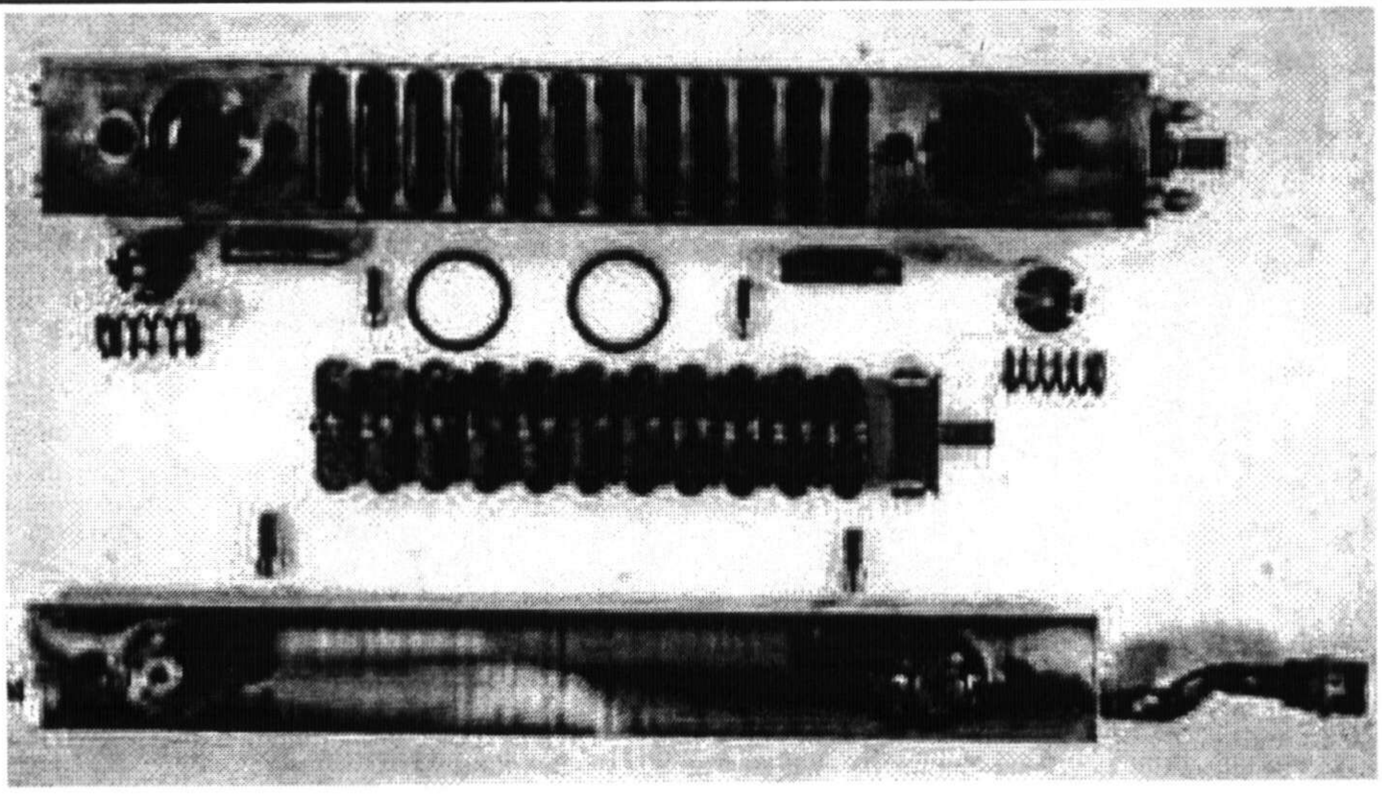
The NX-Borehole Jack (Goodman et al, 1968)







The NX-Borehole Jack



The pistons are in the center section, while the LVDT's are near the extremities. So, the rock-bearing plates may be bent outward at the LVDT locations, creating an excessive displacement.

The NX-Borehole Jack - Data analysis

$$E_{calc} = 0.80 Q_h (D/\Delta D) T^*$$

where

D = hole diameter.
ΔD = change in hole diameter,
ΔQ_h = increment of hydraulic-line pressure, and
T^* = a coefficient depending on Poisson's ratio ν.

T^* *for full contact.*

ν	0.1	0.2	0.25	0.3	0.33	0.4	0.5
T^*	1.519	1.474	1.438	1.397	1.366	1.289	1.151

The calculated modulus, Ecalc, must be corrected as described by Heuze and Amadei, 1985. See also ASTM Standard D 4971-89.

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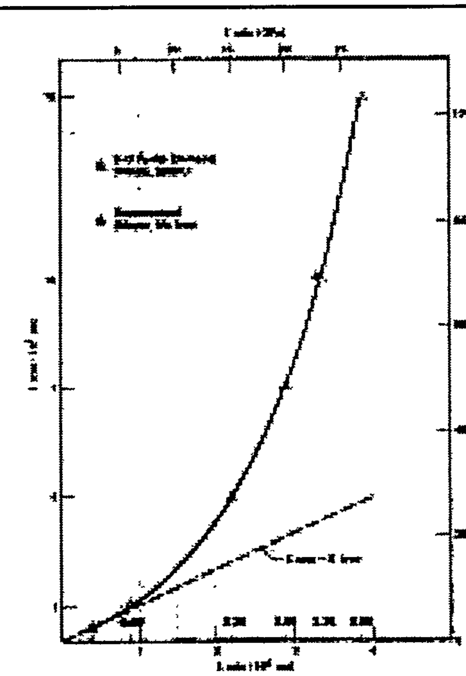
The NX-Borehole Jack - Data calibration

Heuze and Amadei, 1985, and ASTM Standard D 4971-89.



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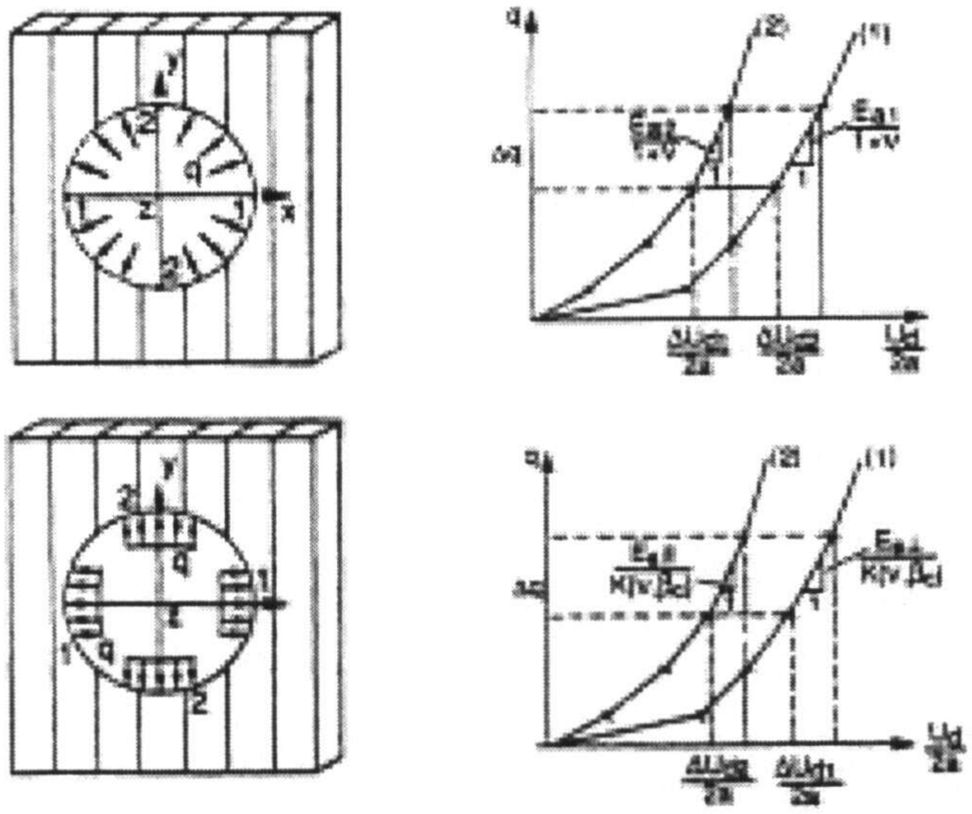
Borehole and gallery tests in anisotropic media

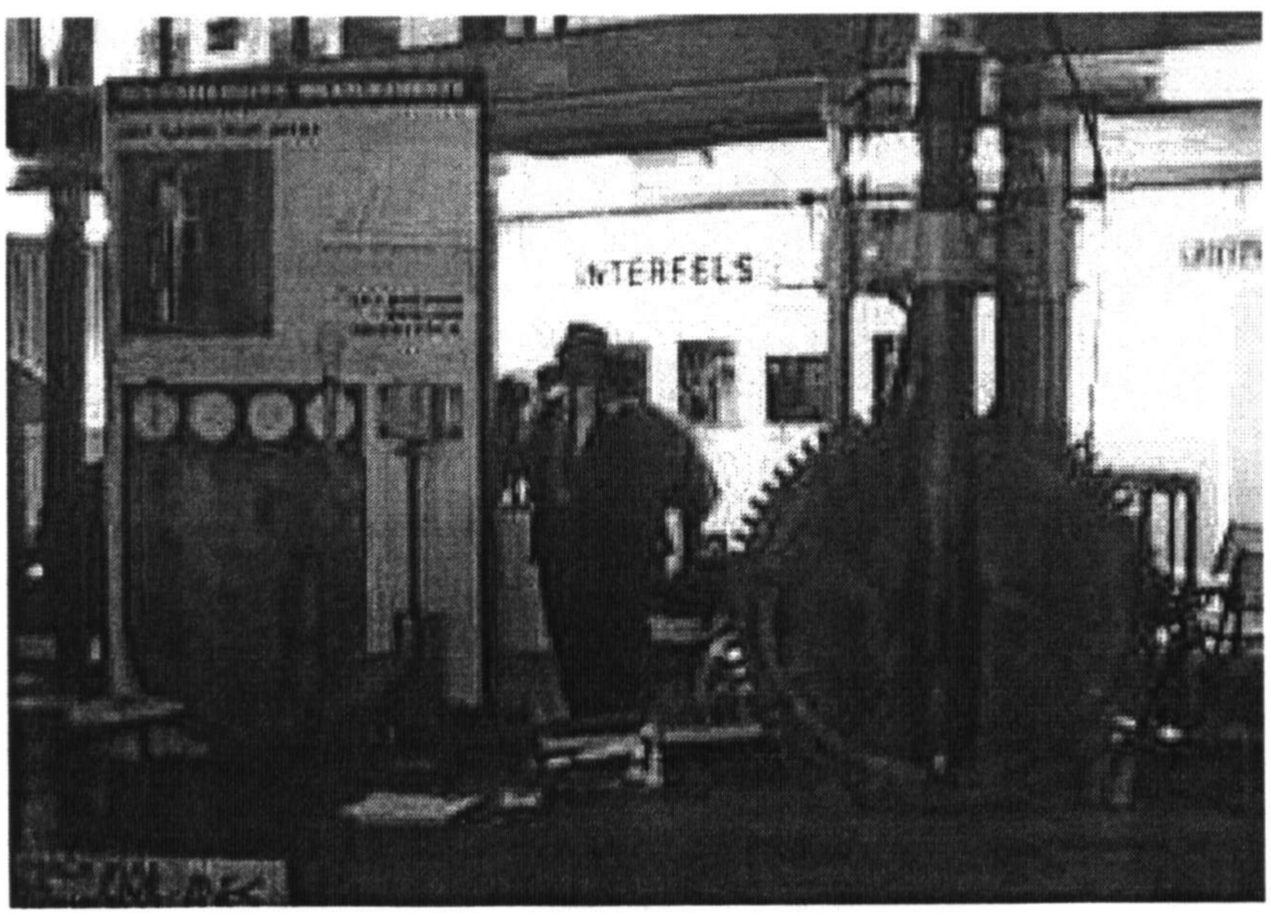
Dilatometer

Borehole jack



(Amadei and Savage, 1991)

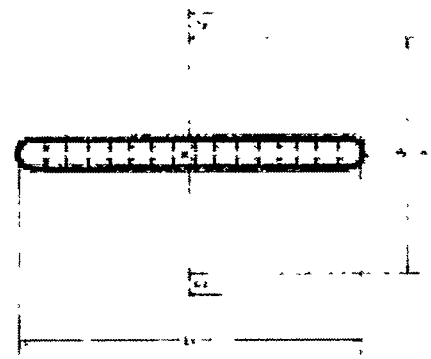
Other field deformability tests - Flat jacks



Flat jacks (cont.)

After Jaeger and Cook, (1976) and Goodman (1980).

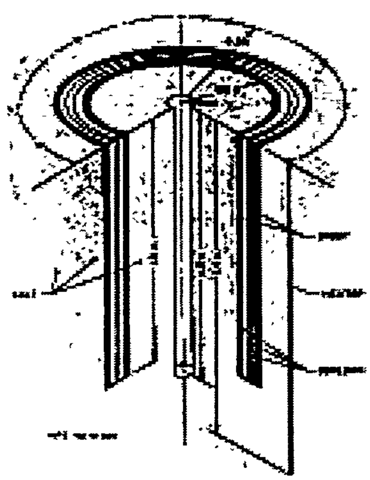
See also Loureiro-Pinto et al , 1986

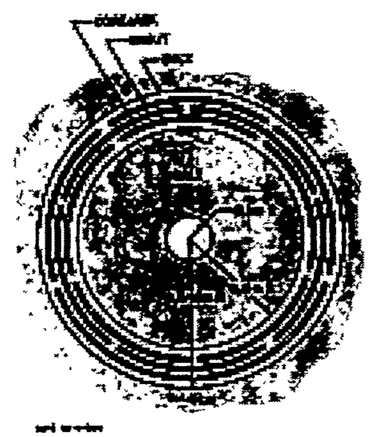


The rock mass modulus E is calculated from the displacement of reference points, upon pressurizing the rock slot with a flat jack

[illegible]

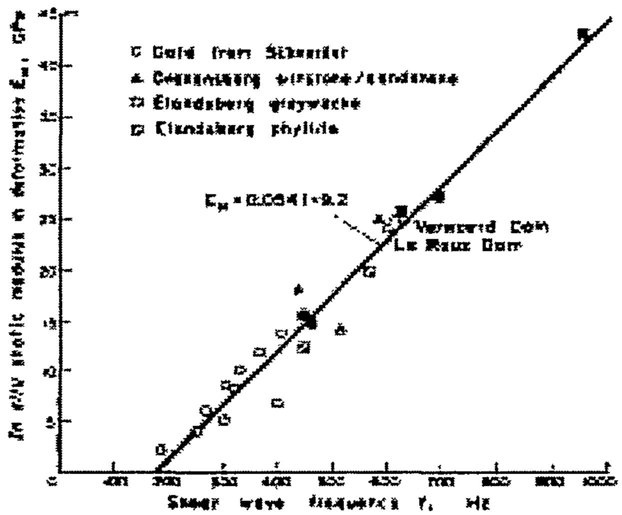
Other field deformability tests - Curved jacks





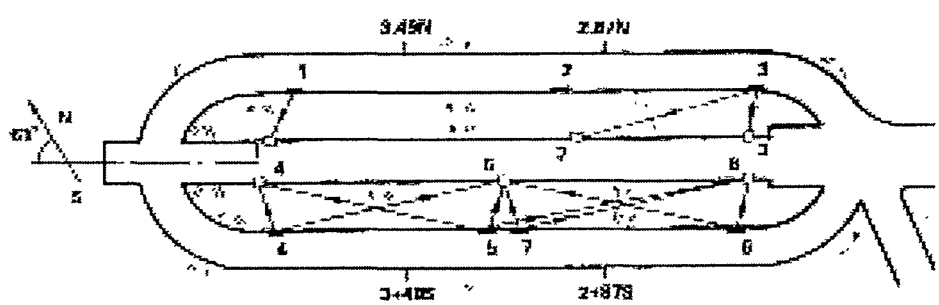
“Corejacking” test in rocksalt (Blankenship and Stickney, 1982)

Other field deformability tests (cont.)



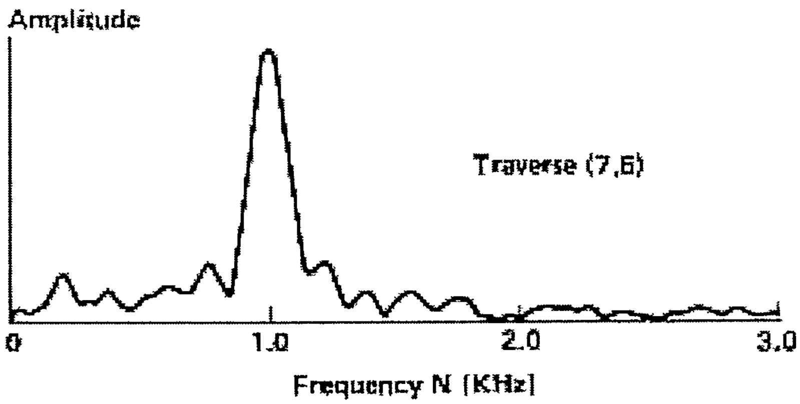
"Petite sismique" results (Bieniawski, 1979). Method proposed by Schneider, 1967.

"Petite sismique" in the Climax granite



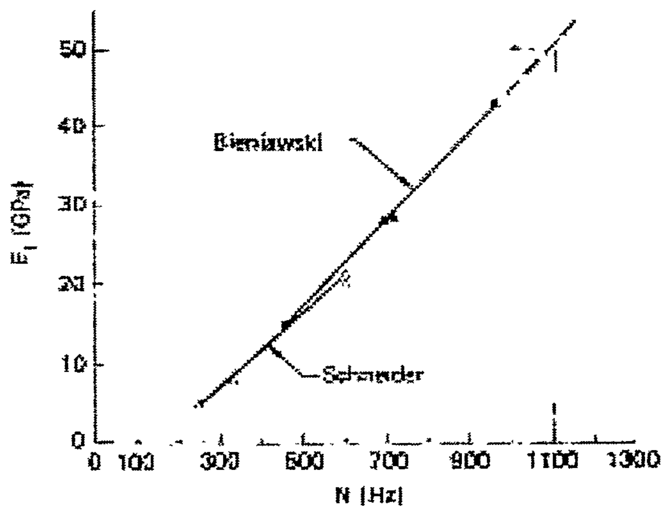
Petite sismique lay-out at SFT-C

“Petite sismique” in the Climax granite



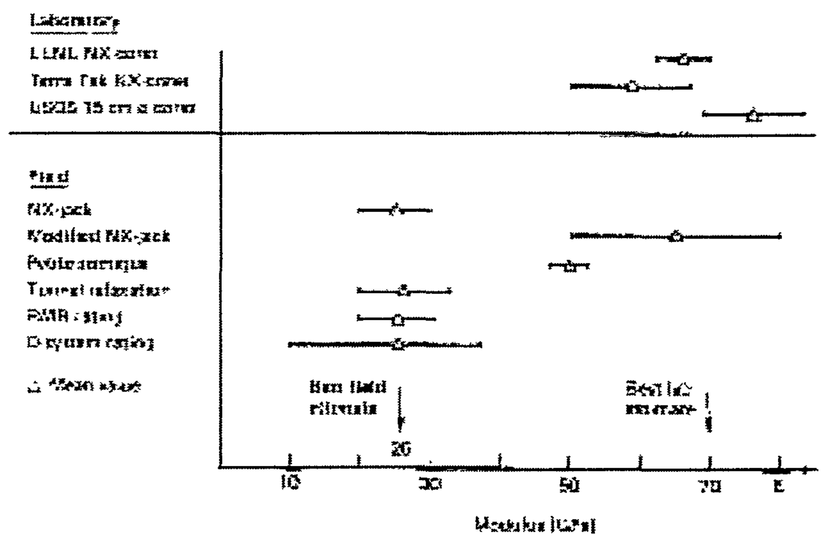
Petite sismique record at SFT-C

“Petite sismique” in the Climax granite



The correlation N-E_{field}, does not seem to fit with other test results or correlations

Comparison of different tests - Scale effects



Climax granite, NTS, Nevada, (Heuze, 1982)

Static vs. dynamic moduli; ex: sedimentary rocks

Dam Name	Country	Rock	Area	E_{stat} (kg/cm^2)	Test	F (kp/cm^2)	E_{dyn} (kg/cm^2)	E_{dyn}/E_{stat}
Sylvenstein	Germany	dolomite	right slope left slope	150 1300	jack load	40-120	73-148	[illegible]
Limberg	Austria	[illegible]	slopes gallery	110-430 300-[illegible]	jack load	6-26	41-1[illegible]	6.2-[illegible]
Speccheri	Italy	limestone	both slopes	520	jack load		600	1
Pieve di Cadore	Italy	limestone	right slope Pian della Era left slope	413 350-415 310	hydr. chamb.	12	35, inj. 52	10-[illegible]
Val Gallina	Italy	limestone	right slope left slope	335 275	hydr. chamb.		60-225, inj. 46 39	7.4-27 43
Vajont	Italy	limestone	upper slopes lower slopes	130-400 114-2400	hydr. chamb.	21 41	41-56 130	0-4 up to 11
Mis	Italy	limestone	valley bottom right slope left slope	170 310 200	hydr. chamb.	30	58, inj. 65, inj.	3.7 4
[illegible]	Italy	limestone	right slope left slope	305 2[illegible]5	hydr. chamb.	55	35 inj.	5.2

The moduli calculated from dynamic tests are generally much higher than those calculated from static tests. In seismic tests, the stress level is usually much lower than in static tests (After Link, 1964).

Static vs. dynamic moduli; sedimentary rocks (cont.)

		E(GPa)			G(GPa)			ν		
Well	Depth (m)	Lab[a] Static	Lab[a] Dyn.	Field Dyn.	Lab[b] Static	Lab[a] Dyn.	Field Dyn	Lab[a] Static	Lab[a] Dyn	Field Dyn
PTS 24-19	1581.6	16.36	45.09	43.62	3.8	23.0	16.66	0.34	0.05	0.31
PTS 22-12	1958.0	16.99	49.50	29.98	6.57	25.12	11.08	0.29	0.024	0.35
PTS 1-10A	3512.5	41.66	66.02	51.12	17.18	33.04	21.04	0.21	0.008	0.21
RR 1-3	3803.6	22.59	47.61	45.21	9.23	35.52	18.31	0.26	0.15	0.28

[a]Values taken to be the average of E, E, G, G, or ν, ν.
[b]Values of laboratory static equals to G, (assuming $G = G$).

3-way comparison of elastic constants for the Mesaverde sandstone

(After Lin and Heuze, 1987)

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Estimating joint normal stiffness at Climax

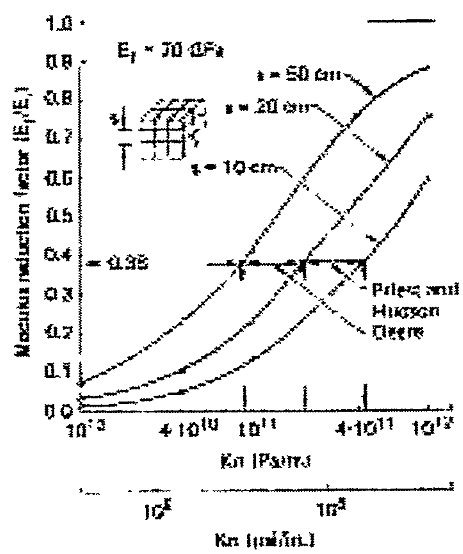
For a rock mass with three orthogonal joint sets, equally spaced, the field modulus is given by (Duncan and Goodman, 1969):

$$1/E_f = 1/E_r + 1/s.K_n$$

where

- **E_r = rock material modulus**
- **s = joint spacing**
- **K_n = normal joint stiffness**

The joint spacing could be estimated from the RQD (next slide).



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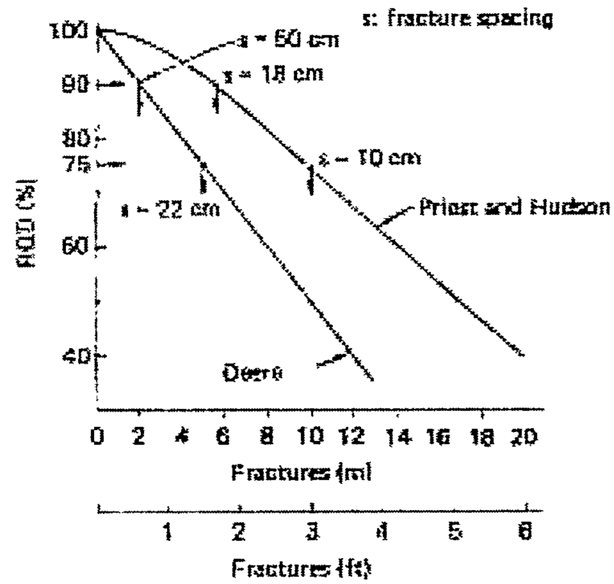
Joint spacing versus RQD

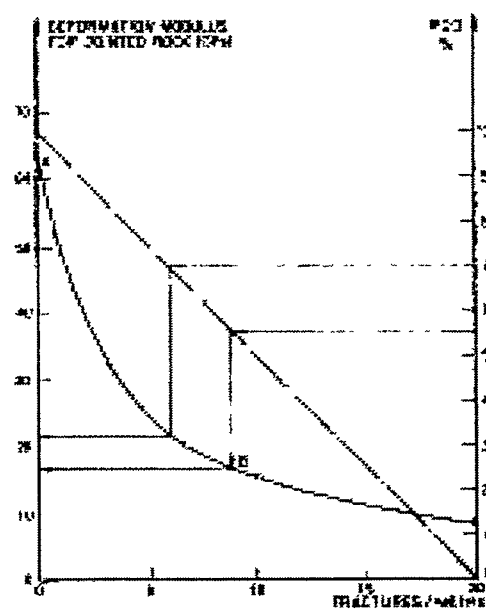
After Deere (1964), and Priest and Hudson (1976)



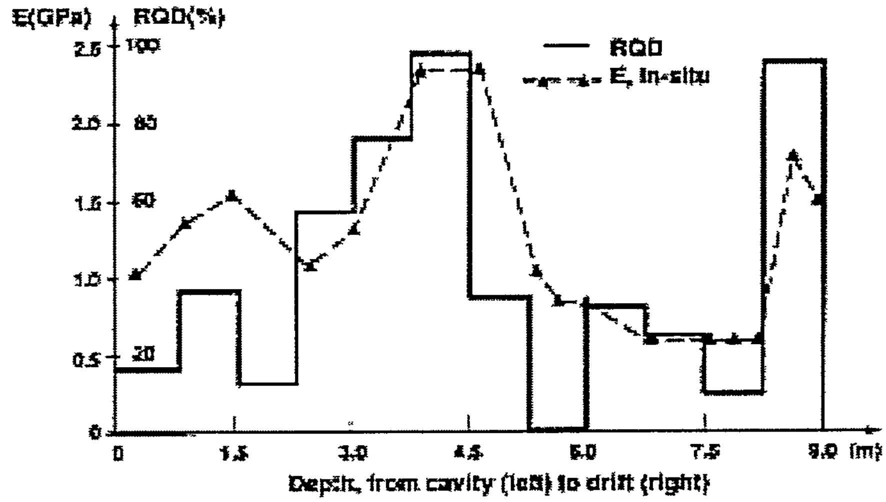
Estimating rock mass modulus variation

In a rock mass with 3 orthogonal joint sets, curve AB can be drawn when knowing the average joint spacing if one knows the modulus at a point or has an estimate of normal joint stiffness ($1/E_f = 1/E_r + 1/s.K_n$).

If the RQD is obtained at another location, the in-situ modulus can then be estimated (Heuze, 1971).



Rock mass modulus versus RQD



Example in tuff , Nevada Test Site, (Heuze et al., 1995)

Additional models of jointed rock masses

$$E_1 = \frac{1}{\left(\frac{1}{E_r} + \frac{1}{S_1 K_{n1}}\right)} \qquad G_{12} = \frac{1}{\left(\frac{1}{G_r} + \frac{1}{S_1 K_{s1}} + \frac{1}{S_2 K_{s2}}\right)} \qquad \nu_{12} = \nu_{13} = \nu_r \frac{E_1}{E_r}$$

$$E_2 = \frac{1}{\left(\frac{1}{E_r} + \frac{1}{S_2 K_{n2}}\right)} \qquad G_{13} = \frac{1}{\left(\frac{1}{G_r} + \frac{1}{S_1 K_{s1}} + \frac{1}{S_3 K_{s3}}\right)} \qquad \nu_{23} = \nu_{21} = \nu_r \frac{E_2}{E_r}$$

$$E_3 = \frac{1}{\left(\frac{1}{E_r} + \frac{1}{S_3 K_{n3}}\right)} \qquad G_{23} = \frac{1}{\left(\frac{1}{G_r} + \frac{1}{S_2 K_{s2}} + \frac{1}{S_3 K_{s3}}\right)} \qquad \nu_{31} = \nu_{32} = \nu_r \frac{E_3}{E_r}$$

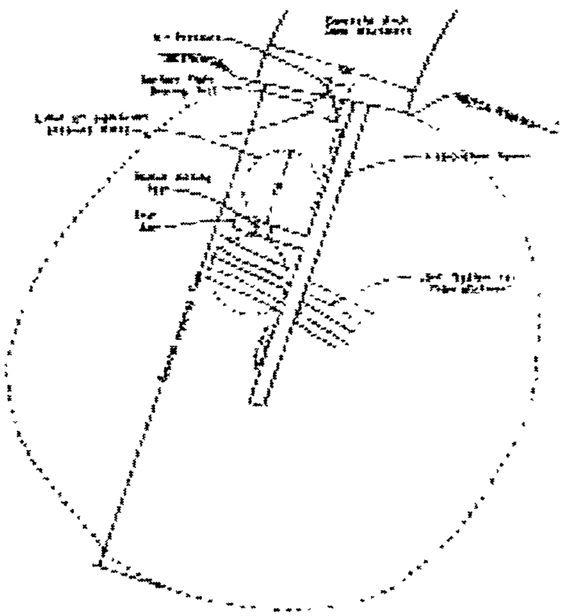
E's : Young's moduli; G's : shear moduli; V's ; Poisson's ratios

Three orthogonal joint sets, not equally spaced (Duncan and Goodman, 1968).
See also Gerrard (1982), and Fossum (1985)

Comparison of different tests - Scale effects

(Wallace et al, 1972)

Different tests will exercise different volumes of the rock mass at different stress levels.



Comparison of different tests - Scale effects

Type of test	"Test Volume"	
	dm^3	ft^3
Strength		
81 cm (32 in) diameter, 2 by 1 cylinder	833	29
1 m cube	1 000	35
15 × 15 cm (6 in × 6 in) plate bearing	170	6
23 × 23 cm (9 in × 9 in) plate bearing	570	20
30 × 30 cm (12 in × 12 in) plate bearing	1 300	48
Deformability		
NX borehole jack	130	4.6
30 cm (12") diameter, plate bearing	955	33.6
91 cm (36") diameter, plate bearing	26 000	900
Pressure tunnel, 1.5 m diameter, 6 m long	40 000	1415
"[illegible]"	Up to several thousands m^3	

These volumes are to be compared to those of the following laboratory tests:

- NX sample ... 246 cm^3 (15 in^3)
- 10 cm (4 in) cube ... 1000 cm^3 (64 in^3)
- 20 cm (8 in) diameter, 2 by 1 cylinder ... 0.013 m^3 (800 in^3)

Heuze, 1980

Summary of scale effects

Heuze, 1980

[illegible]

Summary of scale effects (cont.)

Heuze, 1980

Ratios Em/EL for the Three Rock Classes

Rock class	No. of results	Mean	Std. Dev.
Igneous	15	0.35	0.16
Metamorphics	41	0.36	0.23
Sedimentaries	22	0.42	0.26

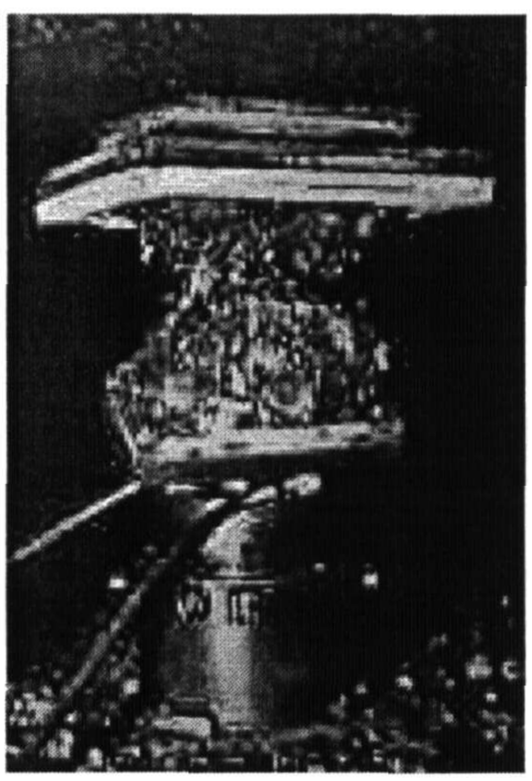
Ratios Em/EL for Various Types of Field Deformability Tests

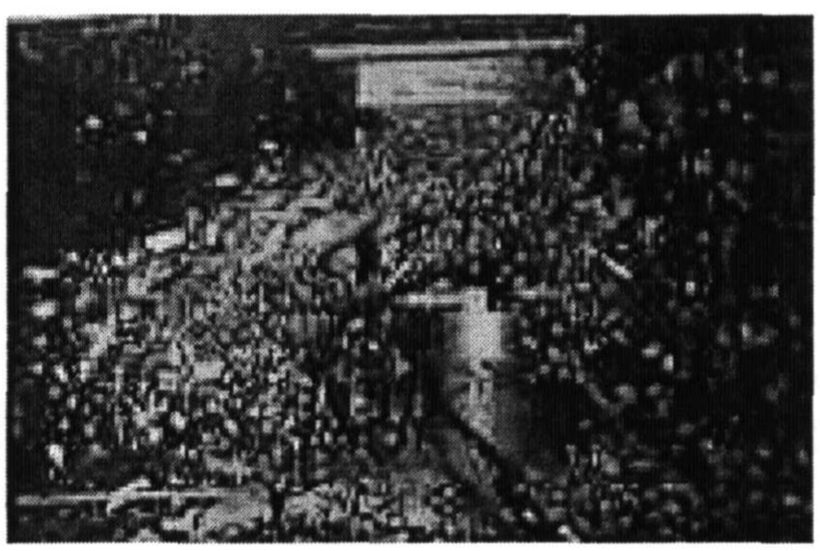
Type of test	No. of results	Mean	Std. Dev.
Plate bearing	27	0.52	0.26
Full scale deformation	14	0.44	0.24
Flat jacks	10	0.54	0.27
Borehole jack or dilatometer	4	0.33	0.17
Pressure chamber	4	0.45	0.22
Petite sismique	3	0.34	0.05
Others	3	0.42	0.14

Part 2

Strength Tests of Rock Masses

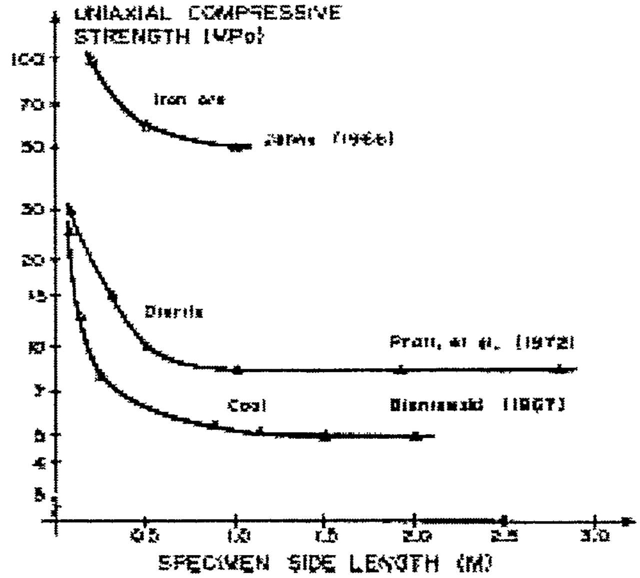
In-situ strength tests - Compressive strength





Bieniawski on coal, 1967. See also Bieniawski et al, 1975

Strength tests scale effects - A reminder



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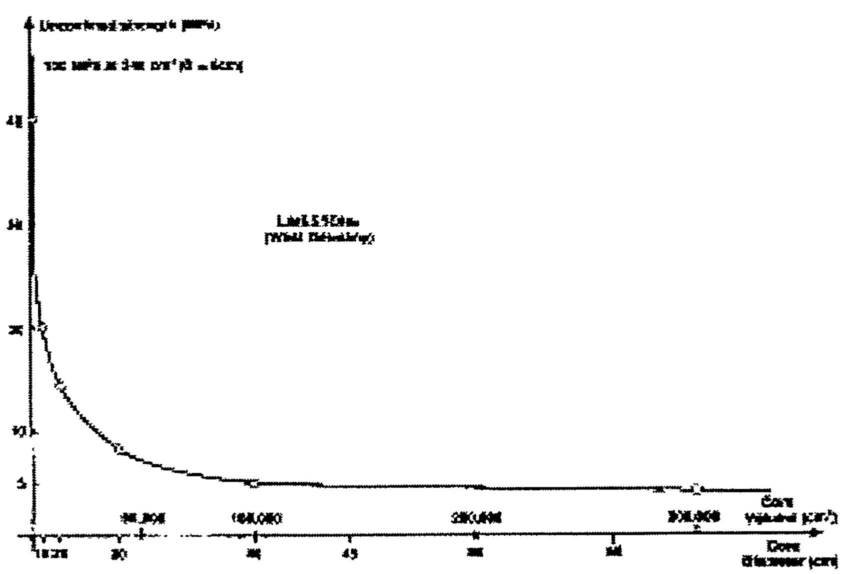
Strength tests scale effects (cont.)



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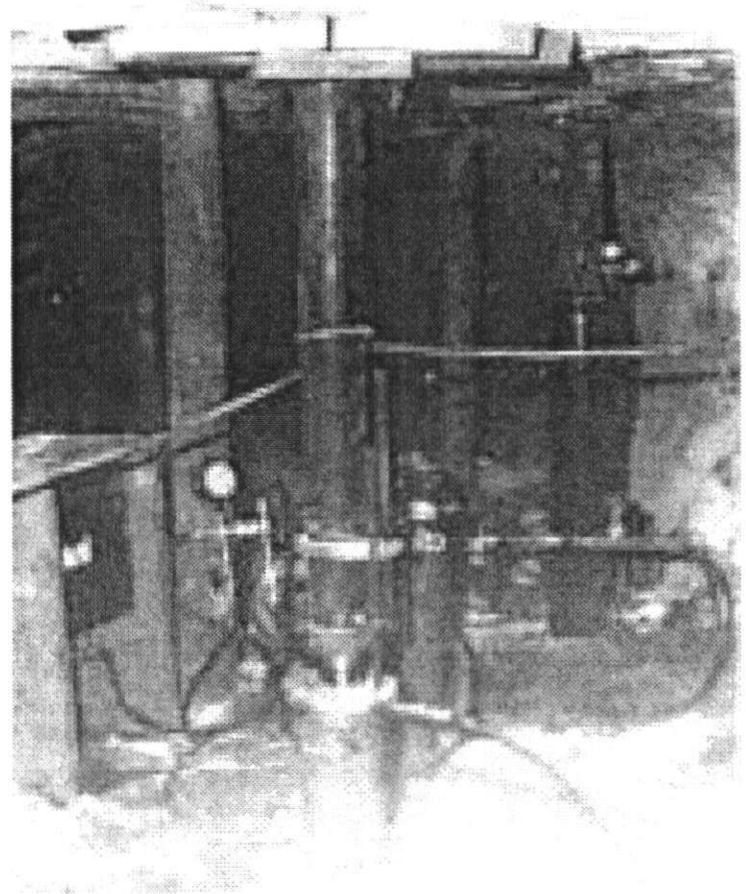
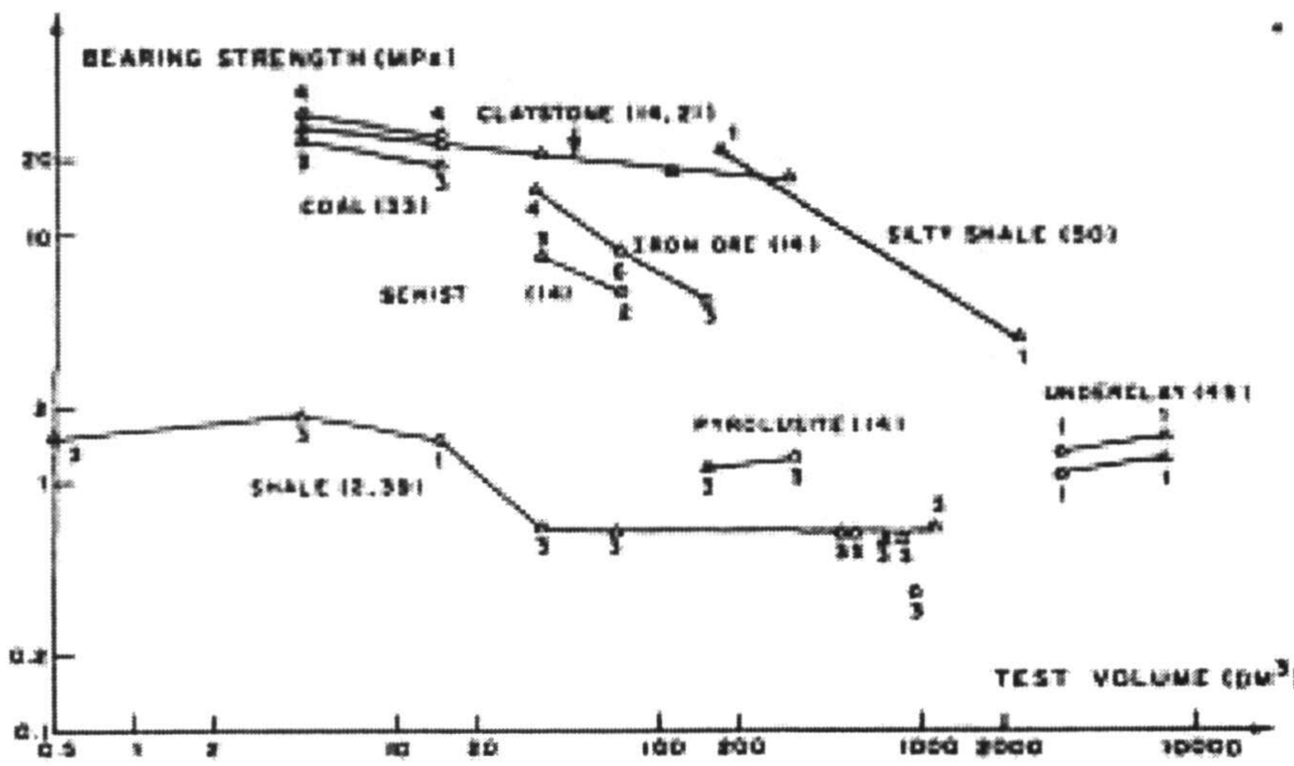
In-situ strength tests - Bearing capacity

Nair, US. Bureau of Mines, 1974



Bearing capacity test results in various rocks

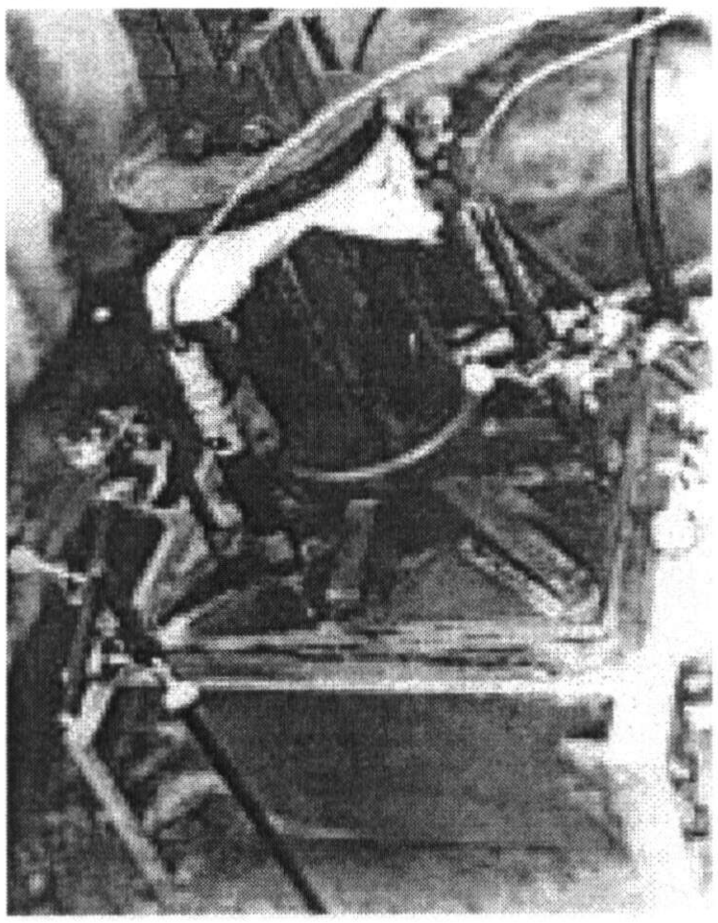


Heuze (1980)

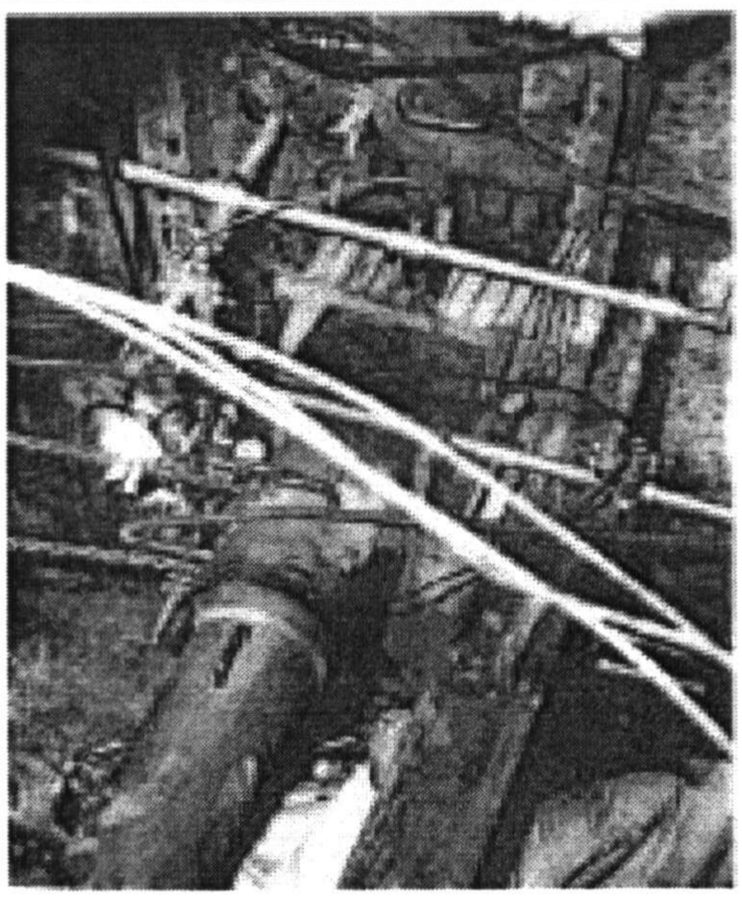
Field direct shear tests - Wallace et al ,1969

Field direct shear tests - Wallace et al ,1969

Field direct shear tests - Wallace et al ,1969



Field direct shear tests - Wallace et al ,1969



Field direct shear tests - another example

Scale effects on joint shear strength

The empirical equation of Barton (1973) for peak shear strength:

$$\tau_p = \sigma_n \tan[\, JRC \log_{10} (JCS/\sigma_n) + \phi_r \,]$$

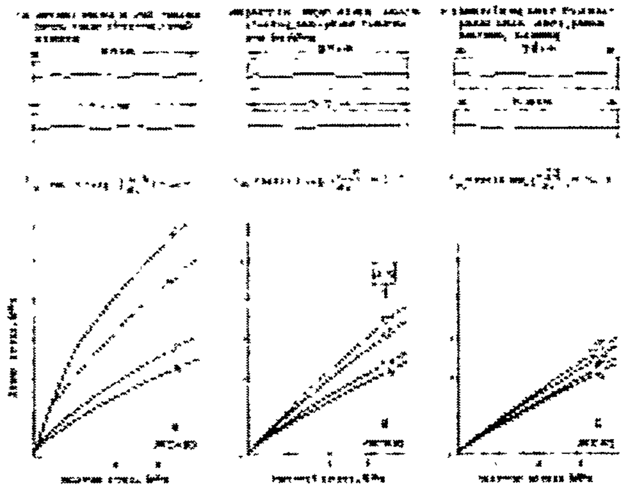
σ_n : normal stress on the joint

JCS : effective joint wall compressive strength (often taken as σ_c)

σ_c : wall rock unconfined compressive strength

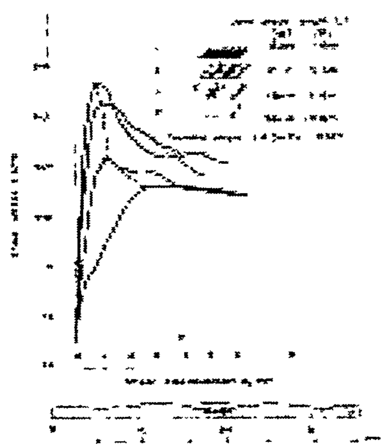
JRC : joint roughness coefficient

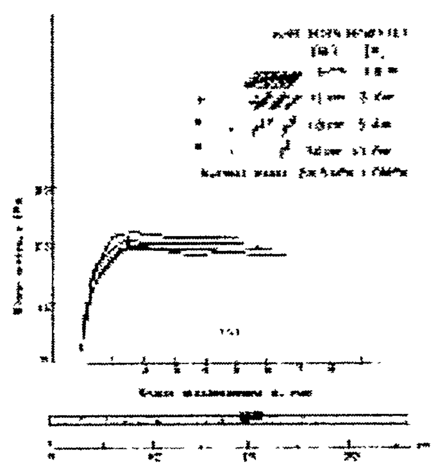
Scale effects on joint shear strength (cont.)



Examples of JRC values and shear strength for different JCS values (Bandis, Lumsden , and Barton, 1981)

Scale effects on joint shear strength (cont.)





Experimental results

Rough joint: scale effect

Smooth joint: no scale effect

Scale effects on joint shear strength (cont.)

Scaling equations proposed by Barton et al, 1985. The subscript n refers to in-situ. The subscript 0 refers to laboratory.

- Shear displacement to peak shear strength. L is the sample dimension in meters.

$$\delta\,(\text{peak}) = \frac{L_n}{500}\left[\frac{JRC_N}{L_n}\right]^{0.33}$$

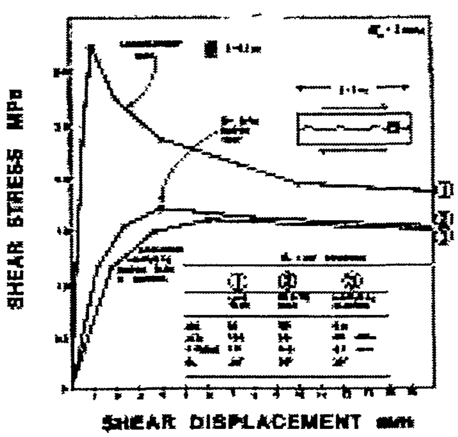
- Joint Roughness Coefficient

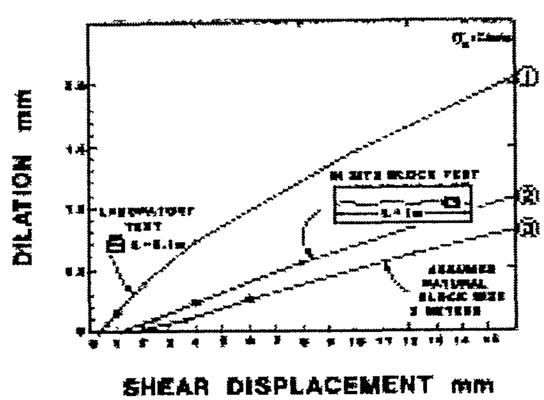
$$JRC_n = JRC_0\left[\frac{L_n}{L_0}\right]^{-0.02 JRC_0}$$

- Joint Compressive Strength

$$JCS_n = JCS_0\left[\frac{L_n}{L_0}\right]^{-0.03 JRC_0}$$

Scale effects on joint shear strength (cont.)





Laboratory results vs. expected in-situ results, based on the preceding scaling equations (Barton et al, 1985)

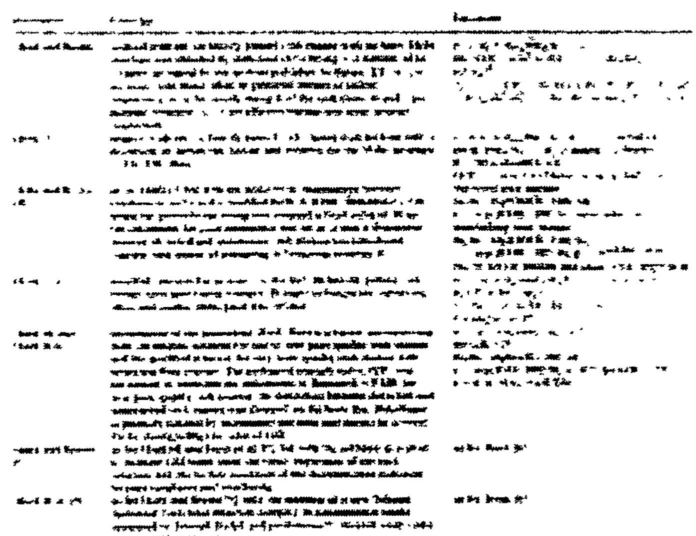
Part 3

Strength Criteria for Rock Masses

- Hoek and Brown criterion -

Overview

Hoek and Brown have produced the best known criteria for estimating the strength of rock masses. Their developments have spanned a period of over 20 years.



After Sonmez and Ulusay, 1999.

The 1980 rock mass strength equation

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m\frac{\sigma_3'}{\sigma_{ci}} + s\right)^{0.5}$$

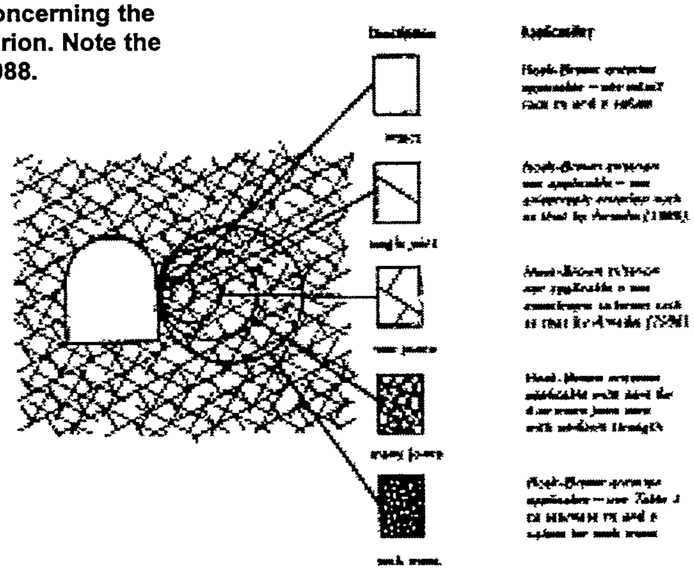
where σ_1' and σ_3' are the major and minor effective principal stresses at failure

σ_{ci} is the uniaxial compressive strength of the intact rock material and

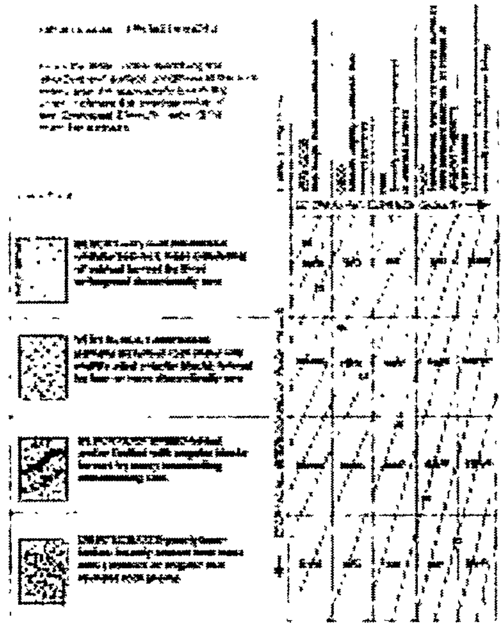
m and s are material constants, where $s = 1$ for intact rock.

The 1988 update

Shows a clarification concerning the applicability of the criterion. Note the reference to Amadei, 1988.



The first GSI (1992)



The 2002 Update

The entire procedure is available online at: www.rocscience.com, in the program RockLab, that includes tables and charts to estimate σ_{ci}, m_i, and the GSI. The strength equations are:

$$\sigma_1' = \sigma_3' + \sigma_{ci}\left(m_b \frac{\sigma_3'}{\sigma_{ci}} + s\right)^a$$

where m_b is a reduced value of the material constant m_i and is given by

$$m_b = m_i \exp\left(\frac{GSI-100}{28-14D}\right)$$

$$s = \exp\left(\frac{GSI-100}{9-3D}\right)$$

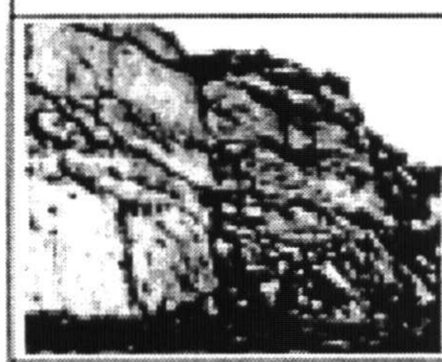
$$a = \frac{1}{2} + \frac{1}{6}\left(e^{-GSI/15} - e^{-20/3}\right)$$

D is a factor which depends upon the degree of disturbance to which the rock mass has been subjected by blast damage and stress relaxation. It varies from 0 for undisturbed in situ rock masses to 1 for very disturbed rock masses

The 2002 Update - The Damage factor

Appearance of rock mass	Description of rock mass	Suggested value of D
	Excellent quality controlled blasting or excavation by Tunnel Boring Machine results in minimal disturbance to the confined rock mass surrounding a tunnel.	$D = 0$
	Mechanical or hand excavation in poor quality rock masses (no blasting) results in minimal disturbance to the surrounding rock mass. Where squeezing problems result in significant floor heave, disturbance can be severe unless a temporary invert, as shown in the photograph, is placed.	$D = 0$ $D = 0.5$ No invert
	Very poor quality blasting in a hard rock tunnel results in severe local damage, extending 2 or 3 m, in the surrounding rock mass.	$D = 0.8$

The 2002 Update - The Damage factor (cont.)

Appearance of rock mass	Description of rock mass	Suggested value of D
	Small scale blasting in civil engineering slopes results in modest rock mass damage, particularly if controlled blasting is used as shown on the left hand side of the photograph. However, stress relief results in some disturbance.	$D = 0.7$ Good blasting $D = 1.0$ Poor blasting
	Very large open pit mine slopes suffer significant disturbance due to heavy production blasting and also due to stress relief from overburden removal. In some softer rocks excavation can be carried out by ripping and dozing and the degree of damage to the slopes is less.	$D = 1.0$ Production blasting $D = 0.7$ Mechanical excavation

The 2002 Update - Empirical modulus vs. GSI

$$E_m(GPa) = \left(1 - \frac{D}{2}\right)\sqrt{\frac{\sigma_{ci}}{100}} \cdot 10^{((GSI-10)/40)}$$

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